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Construction of Cape Peron ocean outlet Perth, Western Australia

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Eutrophication of Cockburn Sound's 100 km^2 of largely land-locked waters lying about 20 km south of Fremantle, Western Australia was well advanced in 1979 when an environmental study confirmed that nutrients from a number of industries and from a major primary wastewater treatment plant operated by the Metropolitan Water Authority were causing extensive algal blooms.[1] In June 1980, the Government initiated several measures to remedy this degradation of the sound's ecosystem. Oceanographic and ecological studies carried out on behalf of the authority throughout 1981 confirmed the environmental acceptability of primary treated wastewater dispersal into well-flushed ocean waters 4·2 km off Cape Peron. Design started in 1982. Land works by MWA's Construction Branch began in January 1983 and included a 250 Ml/day pumping station, 23 km of 1400 mm i.d. effluent pipeline and an oxygen injection station. Contract work for the ocean outlet and transition tower started at the same time. Perth's mediterranean-type climate at latitude 34°S creates conditions conducive to sulphide bacteria corrosion in cement mortar lined pipes. Special protective measures have been designed. Environmental impact constraints on land and marine operations set by the Environmental Protection Authority (EPA) of Western Australia have been met. Offshore trenching through 2·6 km of limestone reefs up to 7 m deep involved blasting and dredging. The outermost 1·6 km of pipeline laid on a sandy seabed in a water depth of up to 20 m, including the 70 port diffuser, was rock armoured. Trenching in surf zones was also rock backfilled. Nineteen 220 m long pipestrings were pulled out consecutively using a 150 t winch barge in a 7-day continuous operation completed on 14 January 1984. All land construction was complete by April 1984 and the project was commissioned for winter flow service in June 1984.

Introduction

The State of Western Australia with an area of 2 525 000 km^2 embraces one-third of the Australian continent with a population of about 1 400 000, 70% of whom live in the Perth Metropolitan region. During the 1950s, the 100 km^2 of Cockburn Sound (Figs 1 and 2) became an outer harbour for the Perth–Freemantle area. By 1975 port facilities mainly for oil, alumina and nickel refineries, a fertilizer plant and a power station occupied 10 km of the sound's eastern shore.

2. The Perth Metropolitan Water Authority decided early in the 1960s to drain all domestic wastewaters south of the Swan River to Woodman Point primary treatment plant, near the northern end of Cockburn Sound. An enlarged

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Ordinary meeting, 5.30 p.m., 3 June 1986. Written discussion closes 17 June 1986; for furtther details see p. ii.

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Fig. 1. Winch barge, block barge and Cape Peron pipeline launching area (Cockburn Sound and Kwinana in the background)

Woodman Point plant was commisioned in 1983 with design average flow of 125 Ml/day by the year 2001.

3. Water-orientated sport and recreation attract up to 15 000 persons to Cockburn Sound on peak summer days. Ecological monitoring first recorded degradation of the pristine ecosystem in the 1960s. Research indicated that the rate of loss of sea grass beds became significant when the total nitrogen loading exceeded 2000 kg/day. By 1979 the loading had reached 2000 kg/day from domestic wastewater effluent plus 3000 kg/day from industry.

4. Domestic wastewater could ultimately add 5000 kg/day at full plant capacity. In 1980, a pre-feasibility study confirmed that continuation of primary treated wastewater discharge into Cockburn Sound was environmentally unacceptable without additional treatment including expensive nitrogen removal. A 12 month feasibility study for an ocean outlet began in January 1981.

Feasibility study and design

Physiography of land and offshore pipeline routes

5. The project lies on the Perth Basin sedimentary deposits which are some 8000 m thick down to the Pre-Cambrian basement rock. Woodman Point Treatment Works stands half a kilometre inland at about 20 m above sea level on the eolianite limestone of Spearwood Ridge. Between this ridge and Cockburn Sound, a flat sandy plain not more than 5 m above sea level extends for 15 km to Cape Peron.

6. There are two parallel coastal limestone formations (Fig. 2). The Garden

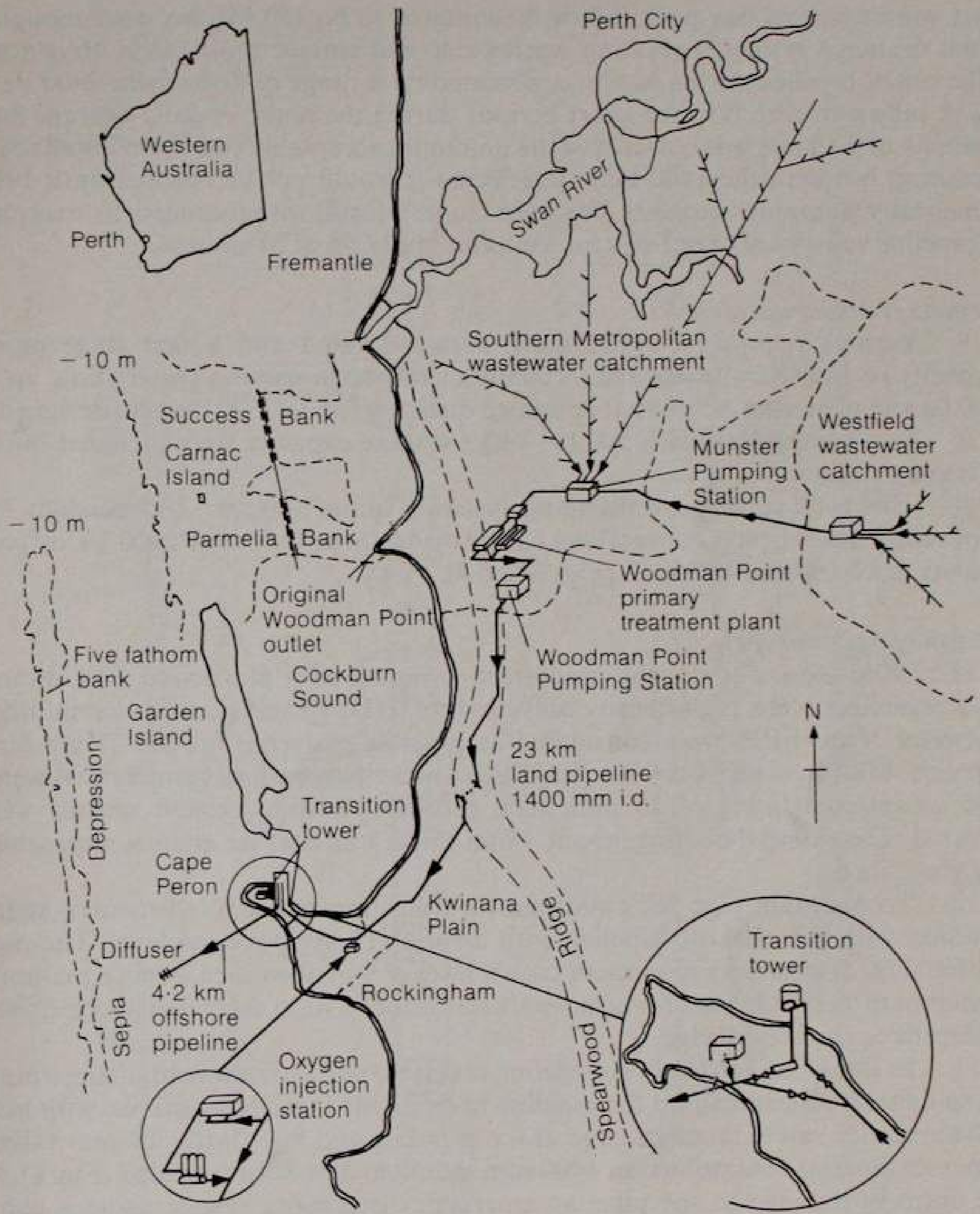


Fig. 2. Schematic presentation of the Southern Metropolitan sewerage system, Perth, Western Australia

Island chain lies 10 km offshore, forming the outer barrier of Cockburn Sound and the coast at Cape Peron. Along the submarine pipeline route submerged ridges, up to 5 m high, separating sand-filled gullies, extend to a depth of 15 m 2·6 km from the shore. Beyond this rocky area a sandy seabed about 5 m thick slopes down to a 20 m depth of water, meeting limestone platform outliers of the five fathom bank 6 km from the shore.

Hydraulic conditions

7. The Southern Metropolitan sewerage area population is estimated to reach over 400 000 by 2001, generating an average wastewater flow of 125 Ml/day. The

wet winter wettest day peak inflow is estimated to be 250 Ml/day even though a dual drainage system separating wastewater and surface drainage is obligatory. The outlet pipeline design had to accommodate a range of flows from 2900 l/s at peak inflow to zero flow for short periods during the night. A daily average flow velocity of 0·45 m/s was considered the minimum acceptable condition for effective scouring but with the 1400 mm i.d. selected it would not be reached until 1990. One daily pumping sequence has been automatically programmed to maintain a pipeline velocity of over 1 m/s for a minimum period of 20 minutes.

Gravity flow and surge control

8. Various pumpset combinations were evaluated and a first stage pump capacity of 1800 l/s was selected. A conjunctive system passing gravity flow up to 800 l/s and provision for ultimate pumped discharge up to 2900 l/s were designed.

9. Two sumps each with 500 m^3 surge storage capacity were included in the pumping station.

10. The total capacity of the sump system is about 1600 m^3. Independent 750 mm reflux valved pipe connections can provide surge inflow at 2000 l/s or pass gravity discharges up to 800 l/s (Figs 3 and 4).

Design of onshore pipe

11. Mild steel, ductile iron, reinforced concrete with plasticized PVC lining, prestressed concrete, high-density polyethylene (HDPE) and glass fibre reinforced polyester resin (FRP) were considered as possible materials for the 23 km land pipeline. Mild steel pipes in 9·3 m lengths, 11 mm thick, with circumferential welds and an internal lining of 25 mm thick sulphate-resisting cement mortar were selected. The external coating, about 4 mm thick, was coal tar enamel impregnating glass fibre.

12. To maintain pipe full conditions, a transition tower was provided at the junction with the offshore pipeline with a sill at 12 m above sea level. The pipe profile (Fig. 5) ensured a maximum crown level of 9 m above sea level to maintain a minimum head of 3 m. To achieve this a tunnel 270 m long had to be driven under the Spearwood Ridge.

13. In designing for onshore pipeline maintenance, environmental constraints on spillage of effluent caused the pipeline to be divided into ten sections, with nine 900 mm sluice valve chambers sited at scour points and fitted with 300 mm valved bypasses. Special vent points on 1400 mm manhole tees were provided (Fig. 6). In the event of damage to the pipe, an emergency bypassing system using a lightweight stoplog in the appropriate manhole tee would enable any damaged section to be isolated and emptied to the sea, within 4 hours of setting up temporary pumping plant.

Control of sulphide bacteria corrosion

14. Perth's extreme mediterranean climate generates conditions conducive to sulphide bacteria corrosion. A vent point monitoring trailer was designed to clear gas pockets, particularly after surge incidents. A 1 mm thick coating of mineral flake impregnated polyester resin was applied to a 1 m width of the pipe crown for about 4 km on areas likely to be temporarily exposed to corrosive atmospheres. The two 15 m deep reinforced concrete pump sumps and the transition tower vortex chamber were lined with plasticized PVC, having dovetailed ribs cast into the concrete.

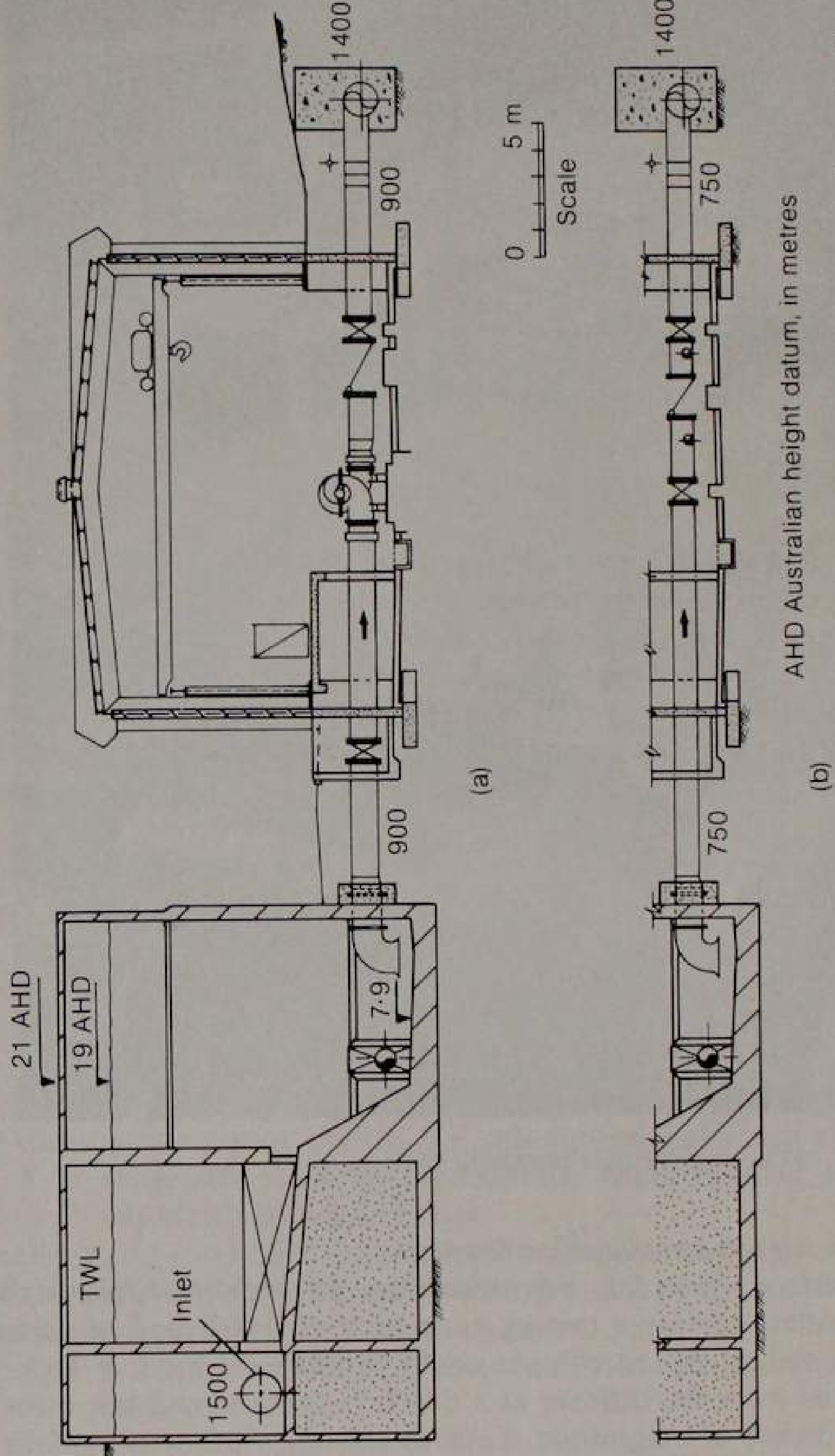


Fig. 3. Woodman Point pumping station: typical cross-section through: (a) pump; (b) surge and gravity flow pipe (dimensions in millimetres)

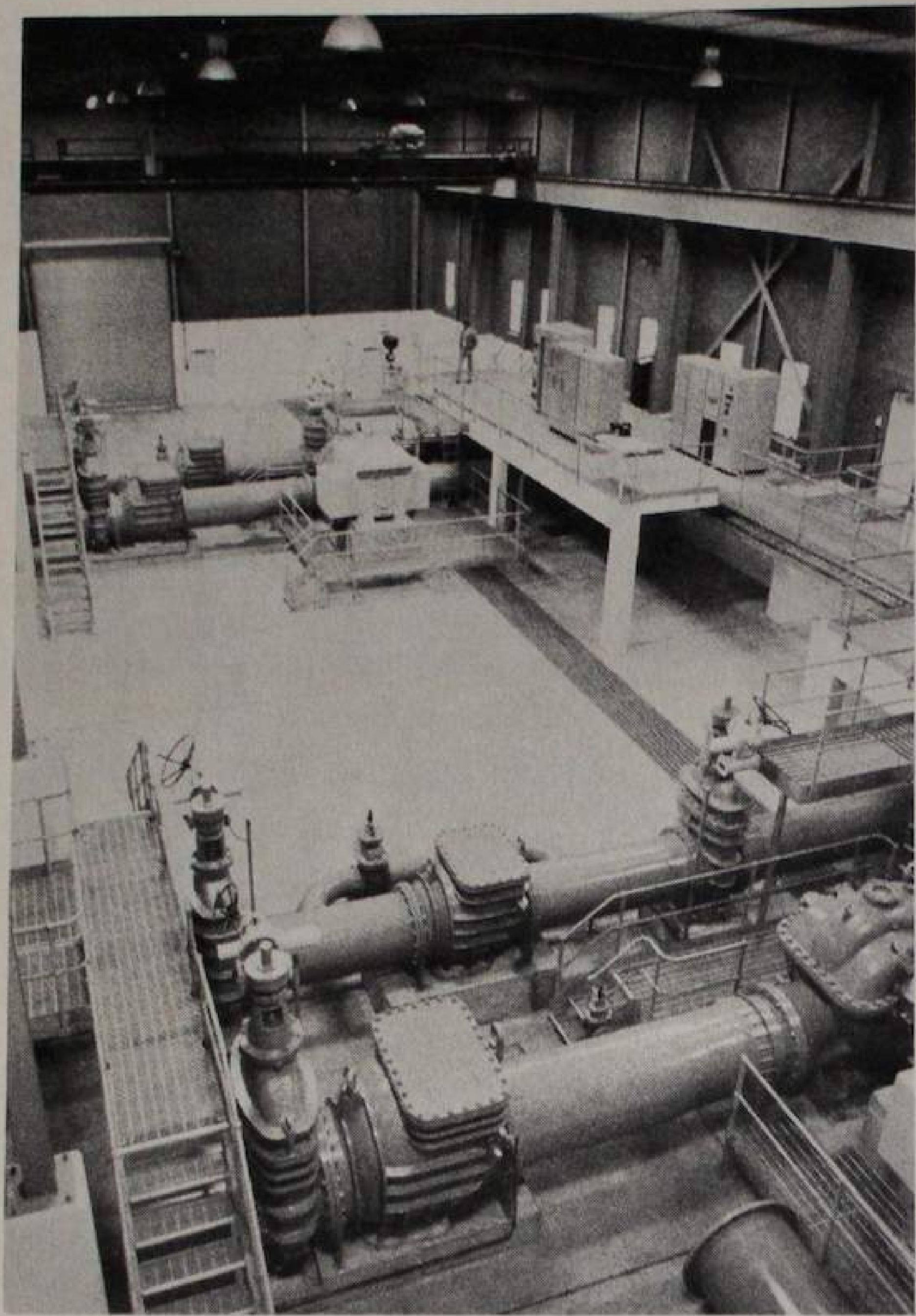


Fig. 4. Woodman Point pumping station

Oxygen injection station and transition tower

15. The Western Australia Environmental Protection Authority's marine water quality criteria specify a mixing zone limit of 0·002 mg/l of undissociated H_2S. Sulphates present in the effluent could produce 20 mg/l of H_2S. The 100 dilutions expected from the diffuser at a depth of 20 m would not meet the H_2S criteria by two orders of magnitude. Tests at existing Authority outfalls indicate that a substantial reduction in H_2S occurs as the effluent mixes with oxygenated sea water. The design provides a curative rather than a preventive approach by treating sulphide generation at an oxygen injection station 21 km from the pumping station. A circulating pump operates a side-stream system using a 710 mm i.d. serpentine dissolver. The oxygen gas flow is controlled by signals from a magnetic flow meter near Woodman Point pumping station.

16. At a point 23 km from the treatment works the effluent enters a vortex

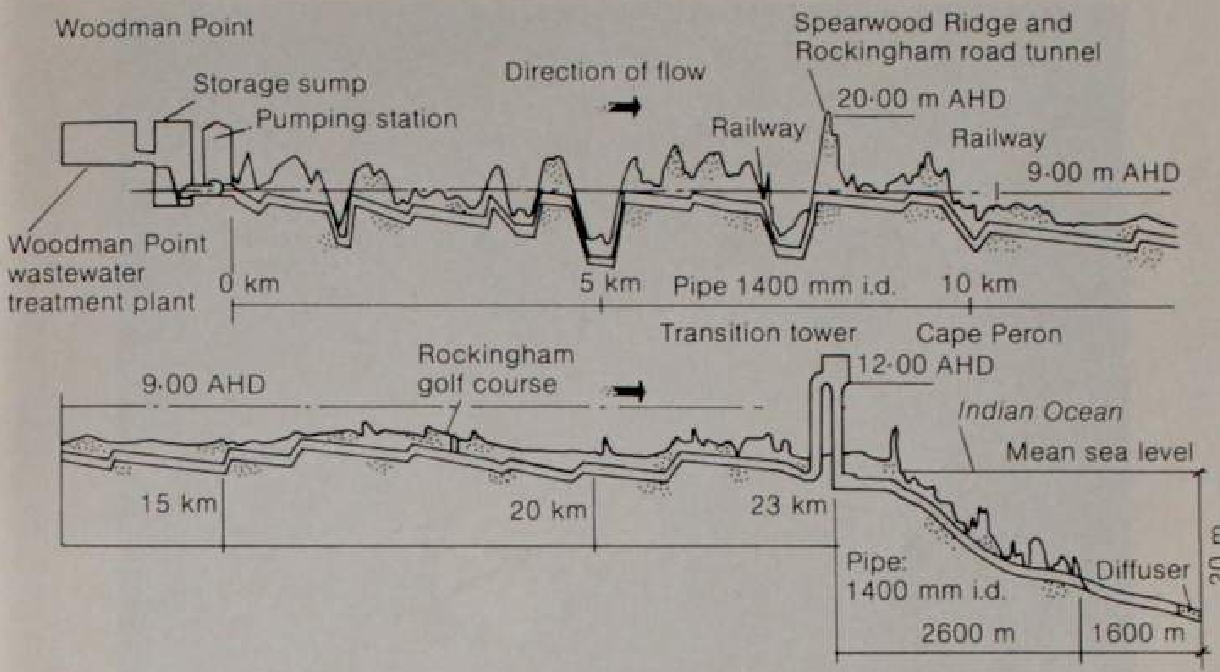


Fig. 5. Pipeline profile

chamber 12 m above sea level and flows spirally down a 1400 mm i.d. vertical drop pipe (Fig. 7). Hydraulic model tests were used to design for minimum air bubble travel into the offshore pipeline. A baffled deaeration chamber, 8 m long, extracts the air which is entrained in the vortex (Fig. 8). Ancillary equipment includes an odour removal unit and a foam suppression spray. For environmental acceptability the tower has been constructed with a fluted concrete surface, and white concrete columns support a spiral staircase (Fig. 9).

Offshore pipeline design

Seabed site exploration

17. A combined echo sounding, side-scan sonar and boomer sub-bottom profiling exercise surveyed a fan-shaped area to select the most suitable pipeline route.[2] A diver-operated submersible drill rig proved the sub-bottom profiling reflectors. The coastal limestone rock can vary from very dense fully calcited caprock, up to a metre thick, to partially cemented strata of shelly sand. The latter tended to disintegrate during coring, but subsequent seabed pit exploration to a depth of 3 m using diver-operated pneumatic tools enabled large limestone samples to be retrieved for rippability tests.

18. In the sandy areas hard surface depths were checked with air/water probes. The risk of liquefaction of the sand foundation under severe wave pressure variations or occasional earth tremors was investigated particularly for the last 1·5 km of the pipeline at depths of 15–20 m. Tests indicated that there was no need for liquefaction prevention measures.

Oceanography

19. Off Cape Peron, there is less than a metre tidal range and currents are mainly wind generated. The Freemantle Port Authority provided 30 years of wind records from an anemometer at a height of 60 m, which were correlated with records from an anemometer installed at Cape Peron at the standard height of 10 m above sea level.

20. A waverider buoy had been operated from 1975 to 1979 about 20 km

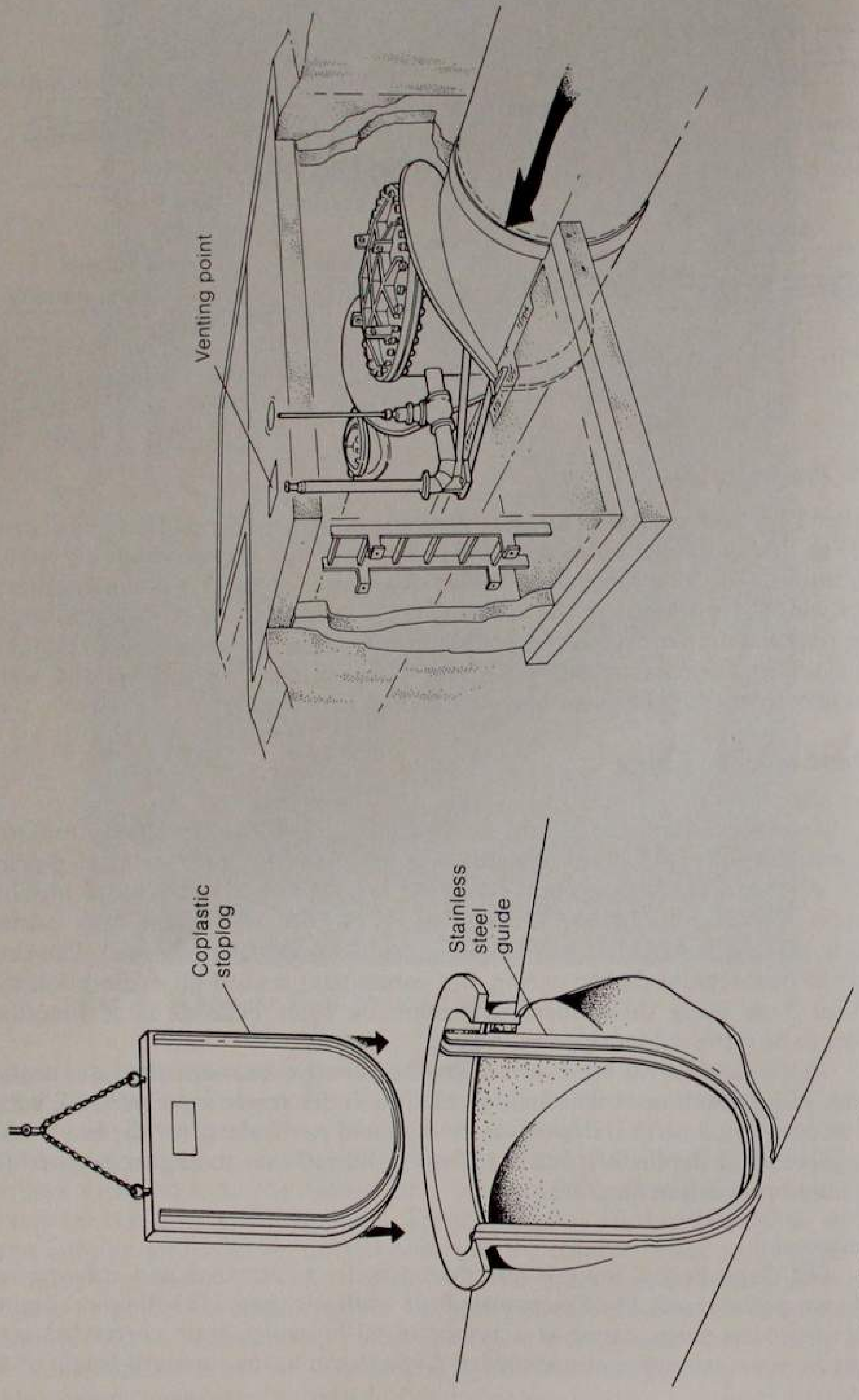


Fig. 6. 1400 mm vent point tee

Fig. 7. Transition tower: one-tenth scale model test of vortex chamber for flow equivalent to 2000 l/s

north-west of the diffuser site offshore of the five fathom bank where the water depth was 40 m. In 1981 this waverider buoy was reinstated and with another waverider buoy at the diffuser location was used to assess the attenuation of storm waves by the five fathom bank. Maximum and significant wave heights were found to be generally reduced by about 40%, with little change in the wave period. A significant wave at a depth of 15 m, of height 4·5 m and period 11 s, approaching the pipeline at 30° to the perpendicular, was taken as the 100-year return period storm condition.

21. During 1981 current meters at six locations produced 54 months records. At the diffuser one meter was at mid-depth (about 10 m) and another 3 m above seabed. EG & G CT/3 electromagnetic current meters were used for pipe stability measurements, complemented by records from the more sensitive NBIS acoustic current meters used for low current speed recording for effluent dispersal investigation.

22. Sea Data 635–12S directional wave recording equipment was installed on the seabed near the diffuser site in June 1981 and operated through the winter months. It measured wave pressure changes and horizontal velocity components 1·5 m above the seabed, and tide heights and water temperature. The results were examined to determine whether orbital current measurements varied from velocities calculated using theory. No significant difference was found.

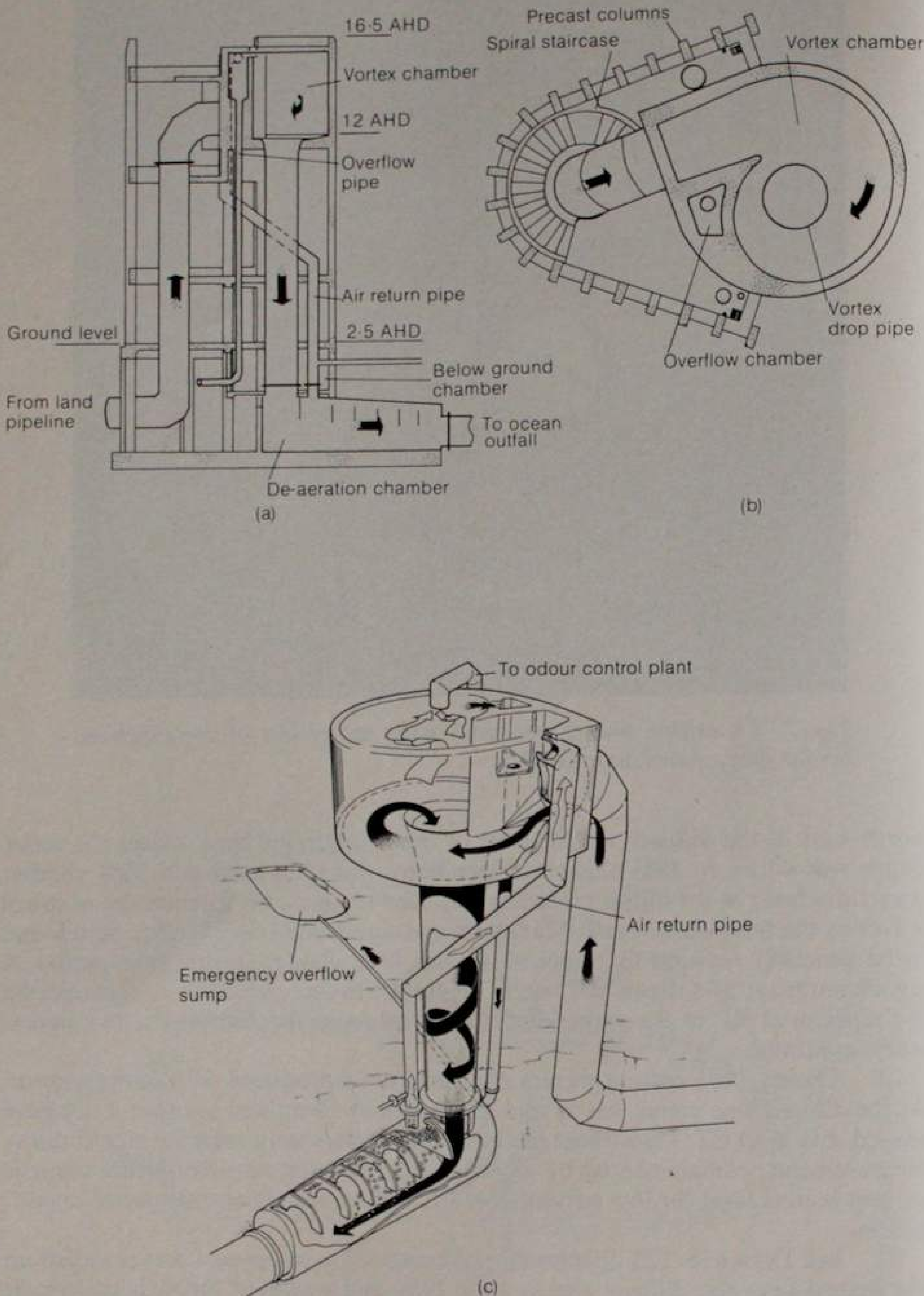


Fig. 8. Transition tower: (a) section; (b) plan; (c) schematic diagram

Fig. 9. Transition tower

Materials and buckling and bending criteria

23. Mild steel, ductile iron, reinforced concrete, HDPE and FRP were considered for the 4200 m of 1400 mm i.d. offshore pipeline. Twin 1 m dia. HDPE pipes were investigated but wall thickness requirements reduced their flow capacity. Mild steel was selected, it had a 25 mm sulphate-resisting cement mortar lining, and was double coal tar enamel wrapped to a thickness of 6 mm, concrete weight coated and cathodically protected.

24. The risk of buckle propagation along an empty pipe with winch pull loading up to 500 t was assessed. For the 1600 m of pipe under a 15–20 m depth of water, an 18 mm thickness of 250 MPa steel was specified with a 16 mm thickness for the remaining 2600 m. The limiting bending radius to protect the 25 mm thick cement mortar lining in a pipe with 16 mm thick steel was specified as 2500 m. The final design of the launchway rail track and rollers highlighted the need to provide

transition curves before and after circular curve tangents, to meet this limiting radius. Safe submerged spans for empty and full pipe were calculated for the 16 mm pipe as 260 m and 30 m respectively. The latter span had to be achieved by sand filling or bagging before the pipe could be finally flooded.

Construction of land pipeline

Administration and design support services

25. The Authority's Construction Branch constructed the pumping station, 23 km of land pipeline and the oxygen injection pumping station (Fig 2 and 10). Steel pipes were manufactured under contract in Perth. Specials were fabricated by the Authority's Mechanical Branch. All mechanical and electrical equipment was supplied under contracts prepared by the consultants and installed by the Mechanical and Electrical Branch of the Authority.

26. The consultants were responsible for all design work on the pumping station, land and offshore pipelines. Technical specifications and bills of quantity were prepared for direct administration work. On the land works, the Authority's registered Materials Testing Laboratory and Quality Control Section carried out testing of concrete, earthworks and pipe manufacture. The consultants were commissioned to give design support services, particularly for special design features related to sulphide bacteria corrosion prevention, surge and cathodic protection measures.

Induced voltage mitigation

27. Excavation was for 11 km in limestone and 12 km in sand. Dewatering was used extensively in sandy areas, with careful monitoring to limit drawdown and prevent saline intrusion of local wells. Laying of 23 km was completed in 14 months. The pipeline ran parallel to high-voltage power lines and towers (Fig. 11), and induced voltage mitigation measures taken for the protection of welders involved temporary bonding to the pipe of welding machines and metal footplates. Permanent induced voltage mitigation measures are being installed. Three impressed current cathodic protection stations have been provided with test points at 1 km intervals along the route and at foreign structure crossings.

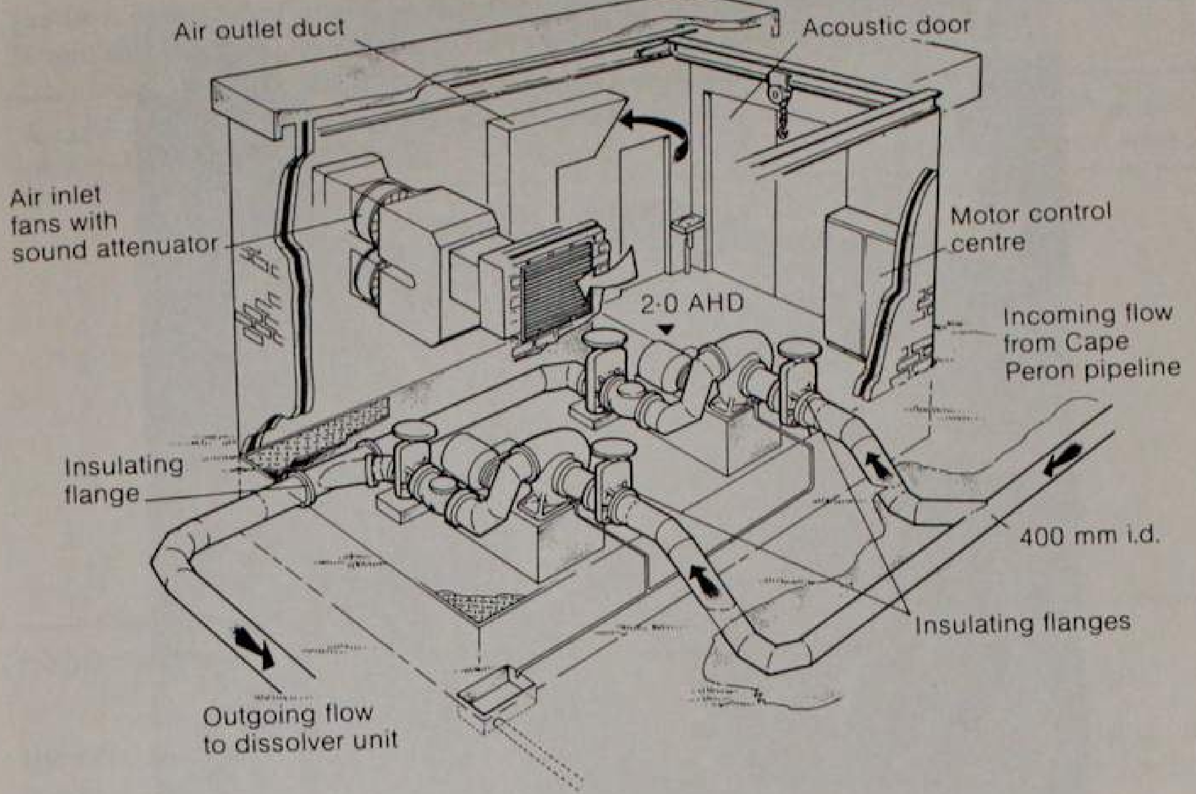
Shrinkage to cement mortar lining

28. Cement mortar shrinkage and steel expansion could result in up to 2 mm total width of cracks in a dry 9·3 m length of pipe when ambient temperature in summer rose to 43°C. The maximum width of individual cracks was about 1 mm, and some penetrated to the steel. A pipe with maximum cement mortar cracking was selected for testing and kept full of water over a period of 14 weeks, with intermittent measurement of crack dimensions with feeler gauges. At the end of the test the pipe was allowed to dry out again in summer heat conditions. Table 1 shows the reduction in crack sizes and Fig. 12 shows one of the test areas. The tests indicated that cracks up to 1 mm wide close to less than 0·1 mm when saturated. Subsequently it is considered that autogenous healing will seal the crack, as described by Parkinson,[3] and shown in his scanning electron microscope photographs (Figs 4 and 5 of reference 3). In view of these tests expensive epoxy filling repairs to cracks were not considered necessary.

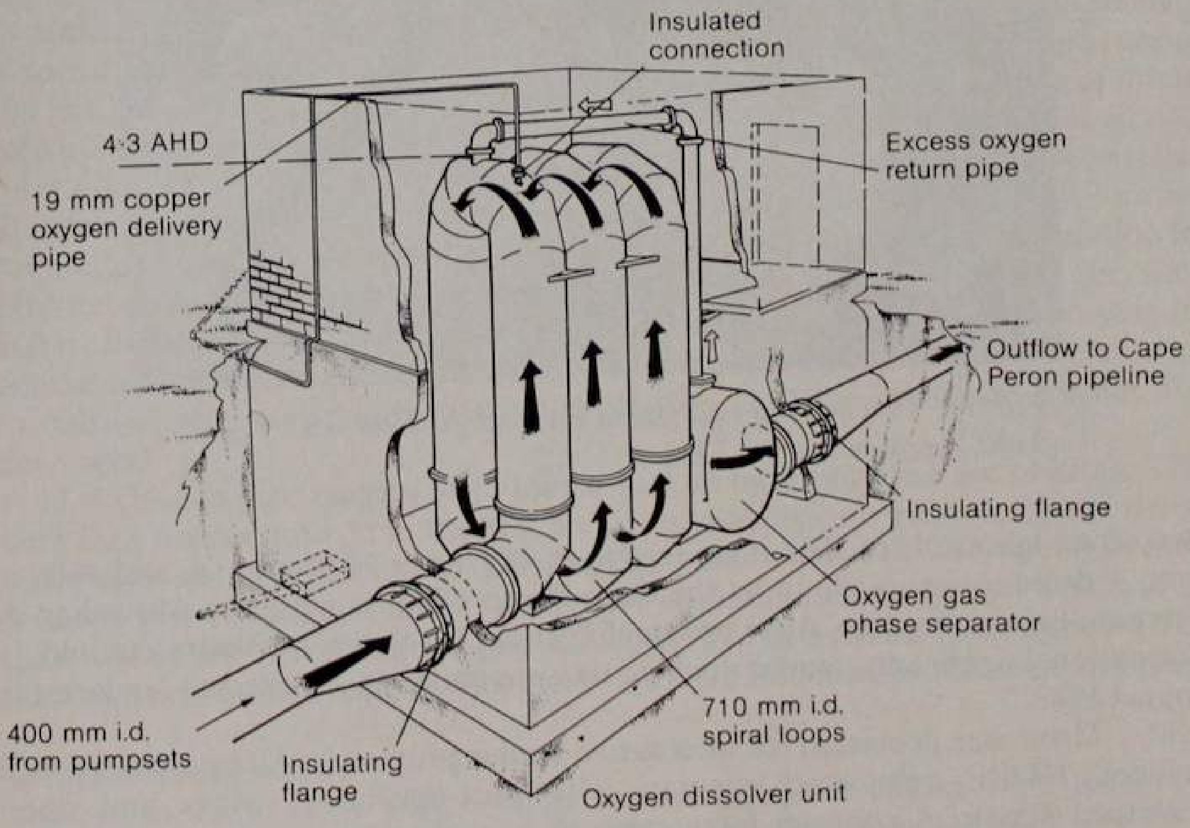
Offshore pipeline

Contract administration

29. The authority had previously constructed four ocean outlets utilizing its



(a)



(b)

Fig. 10. Oxygen injection station (a) pump house; (b) dissolver unit

Fig. 11. Dewatering equipment on land pipeline with Parallel 3300 V power line for 5 km

own direct labour force. The most recent, in 1977, was 1200 mm i.d. and 1600 m long. A decision was made to let the Cape Peron offshore works to Australian or international contractors. After pre-qualification, involving 14 Australian and 15 international applicants, tender documents were issued to six selected tenderers in August 1982.

30. There was provision for alternative tender proposals. Alternatives offered included HDPE, reinforced concrete with post-tensioned strings and fibre-reinforced plastic. A contract for a steel offshore pipeline was let in December 1982, began in January 1983, the pipe was installed in January 1984 and rock armouring was completed by June 1984. The consultant's site supervision team for

Table 1. Measured crack widths (in mm) in a 1400 m dia. mild steel pipe lined with 25 mm thick cement mortar under a test of wetting and drying for 20 weeks

Crack location	Initial width	Crack width* after being wet for 2 weeks	6 weeks	14 weeks	Crack width* after drying out for 6 weeks
1	0·50	0·20	0·10	0·10	0·30
2	0·40	0·20	0·10	0·10	0·25
3	0·60	0·30	0·15	0·10	0·50
4	0·80	0·40	0·10	0·10	0·50
5	0·80	0·30	0·05	0·05	0·50
6	1·00	0·40	0·05	0·05	0·60
7	0·90	0·25	0·05	0·05	0·35
8	1·00	0·30	0·05	0·10	0·40
9	0·70	0·25	0·10	0·10	0·40
10	0·60	0·25	0·10	0·05	0·40
11	0·30	0·15	0·10	0·05	0·20
12	0·30	0·14	0·05	0·05	0·20

* These are surface dimensions. See Fig. 12 for variation at depth of crack.

the offshore contract included four engineer divers, one diving inspector, a quality control consultant and inspector, and two inspectors for dredging and concrete operations.

Fabrication and corrosion protection

31. In New South Wales 2900 t of 16 mm and 18 mm steel plate were manufactured, inspected and shipped to Perth for fabrication. Radiography of 100% of circumferential and 15% of longitudinal butt welds was performed. Achievement of roundness tolerance was critical, particularly to allow smooth spinning at about 200 rev/min during cement mortar lining. A mortar thickness tolerance of minus zero plus 3 mm was specified to assist buoyancy control of the pipes. The manufacturer originally protested that this was half the tolerance allowed by the Australian Standard but was able to meet this stricter specification using care in forming roundness of pipes and control of mortar quality. An impressed current station for external cathodic protection has been installed in the transition tower. The outer 500 m, including the diffuser, where saline intrusion is anticipated, has 50-year-life anodes bolted to the invert for internal protection. Temporary anodes were bolted to diffuser ports to provide external protection until the impressed current field developed.

32. Double pipe lengths were joined first using submerged arc welding. They were then formed into 55 m lengths, and subsequently into nineteen 220 m strings using semi-automatic Lincoln inner-shield welds. 100% of 457 circumferential field welds were radiographed. Twenty-two required minor repair to 0·05% of the total weld run. After radiography the gaps in the external coating were shot blasted, primed and wrapped with a 450 mm wide bituminous adhesive wrapping.

Weight-coated concrete

33. For each 55 m unit, weights and dimensional checks were fed into a computer program to determine weight-coat thickness, nominally 119 mm for 16 mm and 113 mm for 18 mm plate. The original specified target submerged weight was 60 kg/m, but this was subsequently increased to 70 kg/m for the first 3000 m,

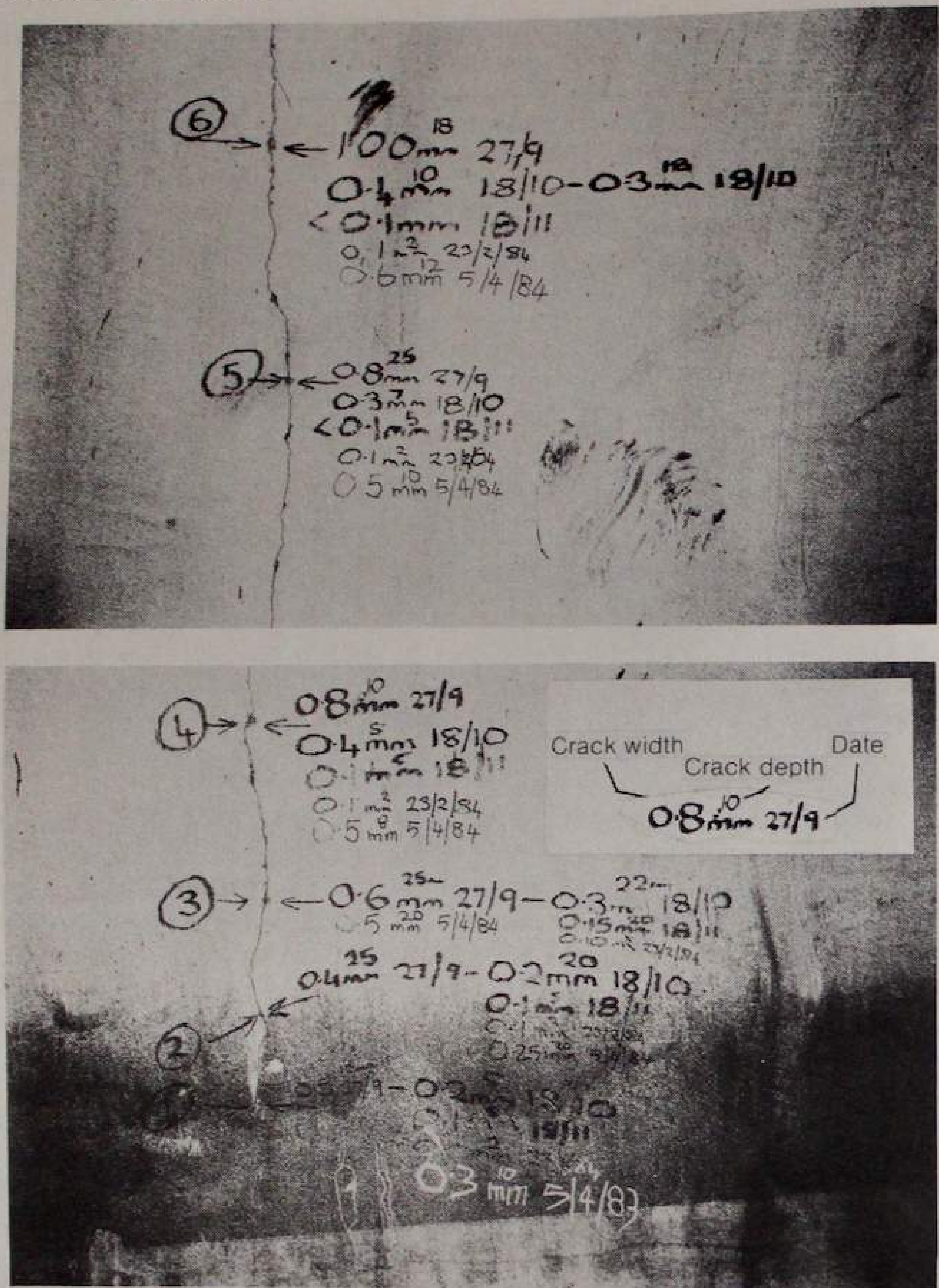


Fig. 12. Cement mortar crack movement test

and 100 kg/m for the remainder. Fig. 13 shows the submerged weights achieved. Two 55 m long steel soffit forms and one circular steel shutter adjustable at the bottom were used (Fig. 14). Seventy-six 55 m weight-coated lengths were cast in 13 weeks, each having 32 m^3 mesh reinforced concrete using diorite aggregate with Daracem super plasticizer admixture. The average mix densities and strengths are given in Table 2. The weight-coated pipe was lifted at 15 MPa strength. The steel form ensured an accurate volume of concrete for negative buoyancy control and a smooth surface for the launching rollers.

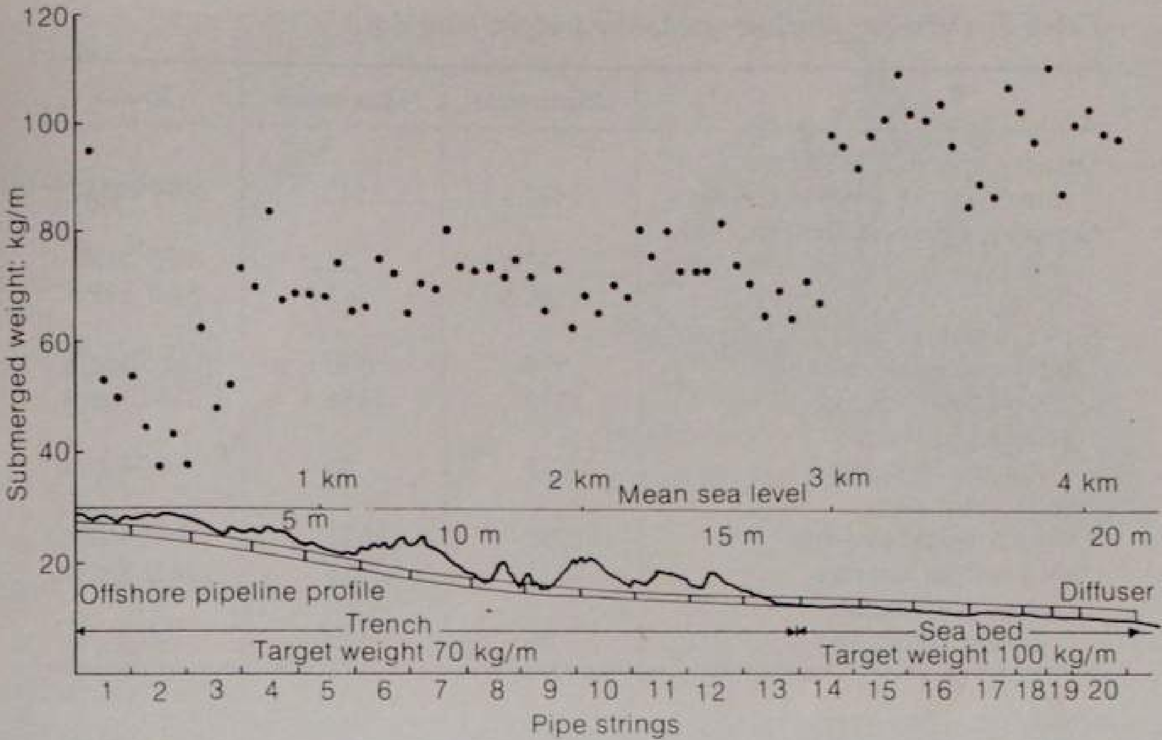


Fig. 13. Offshore pipeline: submerged weight check

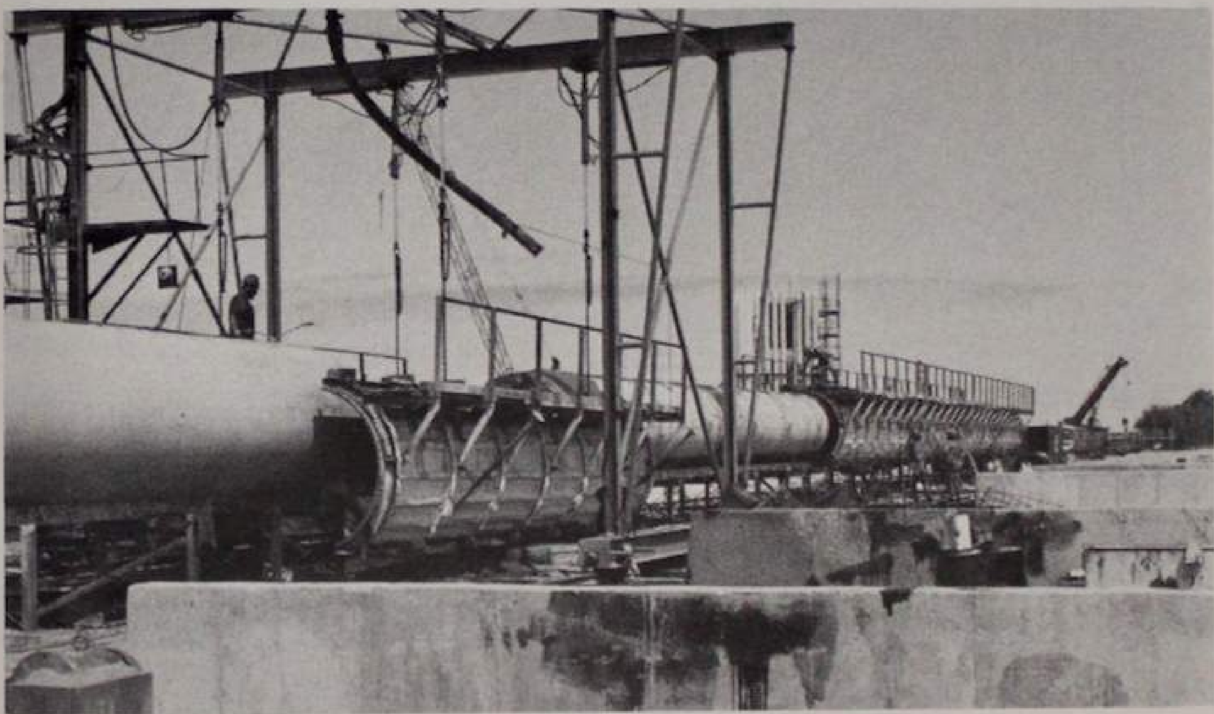


Fig. 14 Steel formwork for casting concrete coating

Pipe string storage

34. The 55 m concrete weight-coated pipe was traversed by five 30 t hydraulic jack trollies on to six load cell installations with capacity for weighing up to 180 t ± 50 kg. Each length was then moved on to the main rail track bogies to assemble a 220 m string. 18 hydraulic jack trollies, in four sets, each with one motorized unit, lifted the 520 t load from the bogies and traversed the 220 m string without bending on to 19 pedestal storage walls using 25 mm thick rubber pads for all seatings (Fig. 15). Spans between supports were limited to 12 m to ensure accept-

Table 2. Offshore pipeline: concrete weight-coat data

	Minimum	Maximum	Mean
Density of concrete after immersion in water for 5 days	2421	2546	2494 kg/m^3
Concrete cylinder strength:			
7 day	27.5	53.0	42·5 MPa
28 day	38·5	63·0	54·5 MPa
Pipe (16 mm plate)			
before weight coating	973	990	982·5 kg/m
after weight coating	2419	2486	2464 kg/m
submerged weight (target 70 kg/m)	38	96	68 kg/m
Pipe (18 mm plate)			
before weight coating	1035	1057	1044 kg/m
after weight coating	2425	2488	2472 kg/m
submerged weight (target 100 kg/m)	86	112	99 kg/m

Fig. 15. Offshore pipeline: string transportation

able flexural stresses on the cement mortar. Cement mortar lining gaps were made good with Hallmark epoxy cement mortar. The external weight-coat gaps were concreted. The ends of each string were matched to a circular machined template and restrained with external octagonal bracing to speed fitting up during the pull operation. The last two strings formed the 325 m length of diffuser in which 69 stainless steel ports were fitted at 9·3 m centres, staggered on either side of the pipe, with a vertical crown port at each end. The ports were cast with an internal bellmouth 135 mm in diameter tapped for an external sealing flange. They were welded into the pipe looking slightly upwards at 20° to the horizontal (Fig. 16).

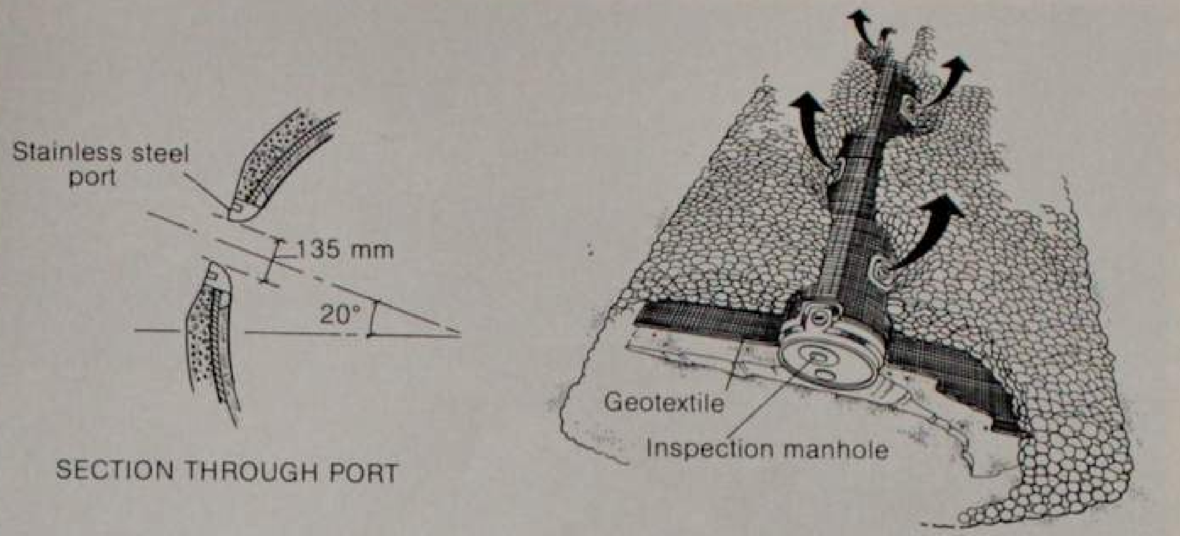


Fig. 16. Diffuser end

Seabed excavation

35. In normal rough weather conditions the surf breaks heavily at a depth of 5 m about 600 m from the shore. A temporary limestone groyne (Fig. 17) was extended for 500 m to a depth of 3 m, and provided sheltered mooring and rock-loading facilities for tugs and barges. The 3 m deep trench alongside the groyne was mainly excavated by two large hydraulic back acters using the spoil to extend the groyne. For the next 2 km intermittent hard limestone outcrops required drilling and blasting. Two barge-mounted grab cranes excavated 80 000 m^3 of sand and broken limestone in 8 months to form a profiled trench with a bottom width of 5 m. Hard rock was excavated to at least 500 mm below the pipe bottom. A 500 mm thick natural sand bedding was specified with a tolerance of plus 500 minus zero. Excavated material was transported in hopper barges to a spoil area. Alignment was maintained by a laser unit, and distance by radio link to a Wild distomat D120 on shore. Accuracy and economy in grab excavation to profile and line was effected by a Mesotech 952 bottom-scan profiling sounder (Fig. 18) used in conjunction with laser, distomat and a tide gauge. The trench in rock areas remained open after excavation but some sand trench excavation was 60% backfilled during winter storms.

Underwater blasting

36. An Atlas Copco ROC701 tracked drill rig, refitted for submarine operation with air compressor installed on the drilling barge, was successfully diver operated in up to 1·5 m swell conditions.

37. Drill holes were loaded with Molanite 110 Watergel explosive in 65 mm dia. plastic cartridges and fired with Magnadet electric detonators. Delay detonators were used for larger blasts to reduce vibration in houses near the coast. About 7·7 t of explosive in 35 blasts were used to break up about 8000 m^3 of hard limestone. The largest quantity used in one blast was 600 kg packed into crevices to break down a rugged limestone outcrop to enable the drill to traverse the area.

38. The use of explosives was monitored by the WA Mines Department. An environmental constraint was put on disturbance to the valuable rock lobster industry during the migration of young lobsters in November and December, called by fishermen 'the run of the whites'. No blasting was permitted from 10

Fig. 17. Temporary groyne

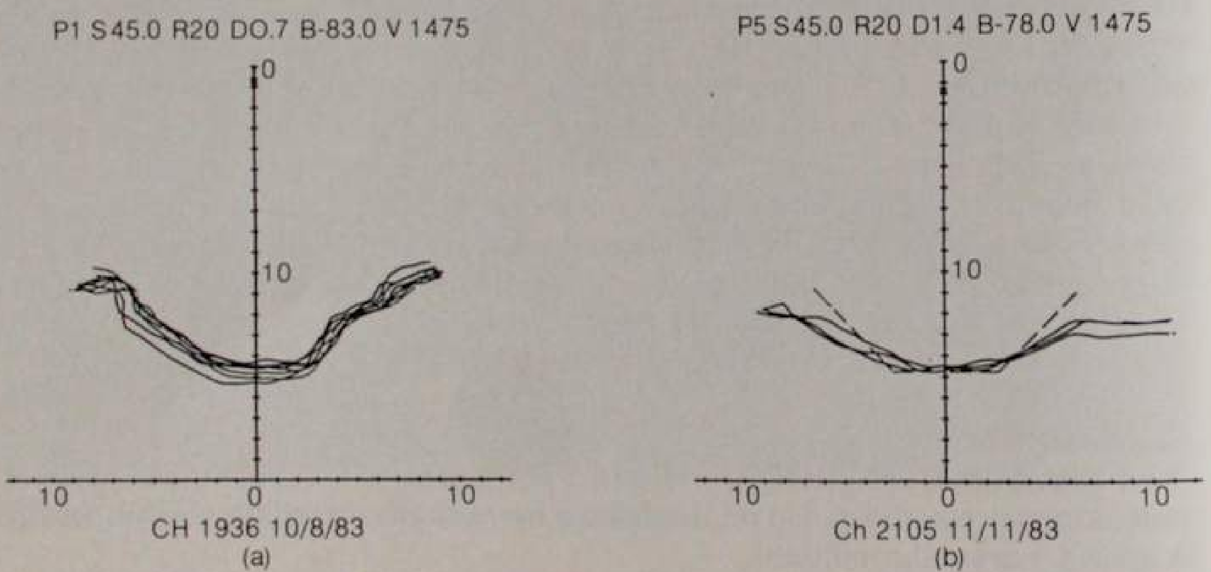


Fig. 18. Mesotech 952 bottom-scan profiling sonar: (a) eight-plot record for detailed check; (b) three-plot record for normal check

November to 31 December.

39. Particular care had to be taken to ensure that all bathers, surfers, skin divers and large marine fauna such as dolphins and seals were not within the dangerous shock wave zone.

Launchway preparations

40. The main rail track was long enough to assemble and test the complete

diffuser. Roller supports then carried the pipe on to the seabed. The entry into the water was arranged in a steel and timber piled trench 5 m wide. 250 × 250 H piles were driven at a spacing that allowed hardwood sleepers to be water jetted horizontally into the sand between the piles. A number of the rollers were suspended from this pile structure using adjustable steel rods to set the rollers to the correct level in the S-curved launchway profile. The 10 t roller ball bearing casings were set in bonded polyurethane tyres. During the operation the polyurethane disbonded on some rollers and subsequent distribution of load to others caused some bearings to collapse. Damaged rollers were successfully reinforced with greased hardwood railway sleepers. A temporary concrete culvert allowed the pipe to pass under the public road (Fig. 19).

Winch pull preparation

41. A pull head, with sledge bottom and two 1400 mm dia. buoyancy tanks on top, was butt-welded to the diffuser (Fig. 20). Air and water connections to the tanks allowed adjustment of pull head inclination. The pull head was designed for 1000 t loading from four 48 mm cable shackles, using two left and two right-hand-lay steel ropes to prevent rotation. An inclinometer was fitted to record any rotation of the pipe.

42. A single anchorage was selected at a point 5·6 km from the shore with four 14 t anchors buried 4 m deep at 15 m centres. 2500 m of 63 mm dia. rope was laid from each anchor to take the final pull loads. An Australian 150 t hydraulic drum winch was tested to 150 t pull on land and then mounted on the largest barge (Fig. 21). A standby 50 t Skagit winch and cable reeling equipment were also installed. The drill barge was refitted to become a block barge between the winch barge and the pullhead. This enabled reeving of the main winch rope to be done on deck and maintained cable catenaries clear of the sand. A 90 t Skagit winch was mounted on the block barge. A load cell was fitted at the winch cable termination for two-part reeve on the winch barge and for three part reeve on the block barge.

Fig. 19. Piled trench and road crossing

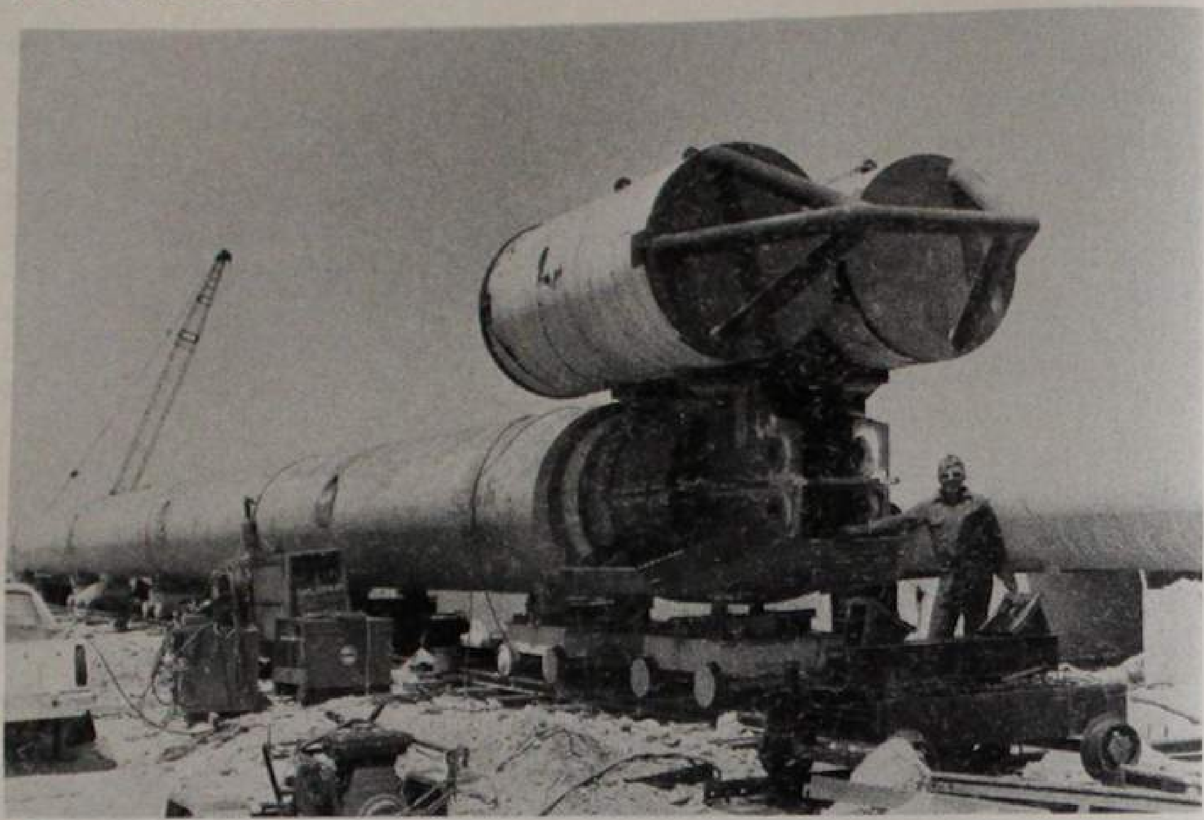


Fig. 20. Pull head and buoyancy tanks

Fig. 21. Winch barge

43. A hold-back winch of 25 t capacity and hydraulic clamp were provided. However, after the first two pulls it was found to be unnecessary.

44. Steel cables were threaded through plastic pipe ducts in the weight coat concrete as slings for ten buoyancy tanks. These were fabricated ready for use in case the pipe became embedded in sand, particularly after emergency flooding. They were not required during the pull.

Weather forecasting

45. Negative buoyancy for the last 1200 m of pipeline exposed on the seabed at a depth of 15–20 m was increased to 100 kg/m. However, the empty pipe would still become unstable when 1 m high waves approached at 90°. Strong onshore breezes in hot January weather can generate significant waves 1·5 m in height, of period 5 s.

46. Fourteen January storms had been recorded in the previous 18 years. Tropical cyclones are prevalent off the north-west coast of Australia in January, with some risk of moving south to Perth. Cyclone precautions were clearly laid down. These were: blue alert, flood the pipe; yellow alert, take marine plant to shelter; and red alert, sink barges on the seabed.

47. A Melbourne Bureau of Meteorology weatherfax printer gave nine regional weather forecasts per day; ocean-routes shipping weather service gave 3-day forecasts by telex; and Perth Bureau of Meteorology gave daily forecasts of local wind speeds and directions. An anemometer, waverider buoy and current meter measured actual site data.

48. Before the pull operation started the conditions for emergency pipe flooding were agreed. At Beaufort force 6 strong breeze exceeding 25 knots the pull-head buoyancy tanks would be flooded and barges put on storm anchorage. When a Beaufort force 7 near gale was forecast, with wind speeds expected to exceed 35 knots for 12 hours from south-west or north-west quarters, the pipe was to be flooded.

49. Darbyshire and Draper hindcasting technique[4] was used to estimate wave heights from wind data. Attenuation, refraction and swell were considered in assessing wave conditions entering the coastal zone. Steady and wave oscillatory current velocities were then estimated to determine the hydrodynamic forces likely to act on the pipe.

Pipe stability during the pull operation

50. During tender negotiations the use of temporary piles for lateral restraint had been agreed, but subsequently the method was changed to avoid risk of fouling cables or pull head. As an alternative, ten hinged collars, incorporating rollers, with 20 t anchorage on either side of the exposed pipe at 150 m centres, were fabricated. These could restrain pipe movement under a 2 m significant wave height of period 7 s acting at 90°.

51. All anchorages were laid, but the last 1000 m of pull proceeded quickly, with safe weather forecasts, and only one collar was placed. A 200 mm water main capable of charging the offshore 1600 m of pipe in 5 h was laid with several offtakes along the launching ramp for emergency pipe filling.

The pull operation

52. After being hydrostatically tested, the diffuser was launched into the trench by adding two strings and pulling the pipeline with large tractors. The pullhead

buoyancy tanks were then fitted by divers about 300 m from the shore and the four cables from the block barge were connected.

53. The winch barge began pulling at dawn on 7 January 1984, and the final pulling operation was completed 2 h before dawn on 14 January 1984. The contractor worked round the clock during the period of the launching operation.

54. At the winch drum speed of 8 m/min, a two-part reeve pulled a string in under 1 h. Divers rode on the pullhead to adjust the buoyancy tanks and watch for pipe rotation, and a maximum tilt of 2° was recorded. The final tilt was less than 1°.

55. The tapered trolley wheels caused gradual spreading of the launchway rails and several derailments occurred. However, the problem was remedied by fitting tie rods between the rails. String welding was a key operation, and four welders had practised simultaneous welding. Accurately rounded ends speeded the fit-up time and the average weld time was 95 minutes.

56. The radiography was carried out as soon as the weld had cooled sufficiently, and following approval by the quality control inspector, the joint was blast cleaned and primed. A special Serviwrap adhesive wrapping was applied that could resist the heat generated by the magnesium phosphate concrete used to fill the weight-coated concrete gap.

57. After about 30 minutes this concrete was tested for 15 MPa strength to check that it was hard enough to cross the rollers.

58. The shortest overall pull operation was $6\frac{1}{2}$ h. After the thirteenth pull, the winch reeving was changed to three-part. The winch had not been designed for

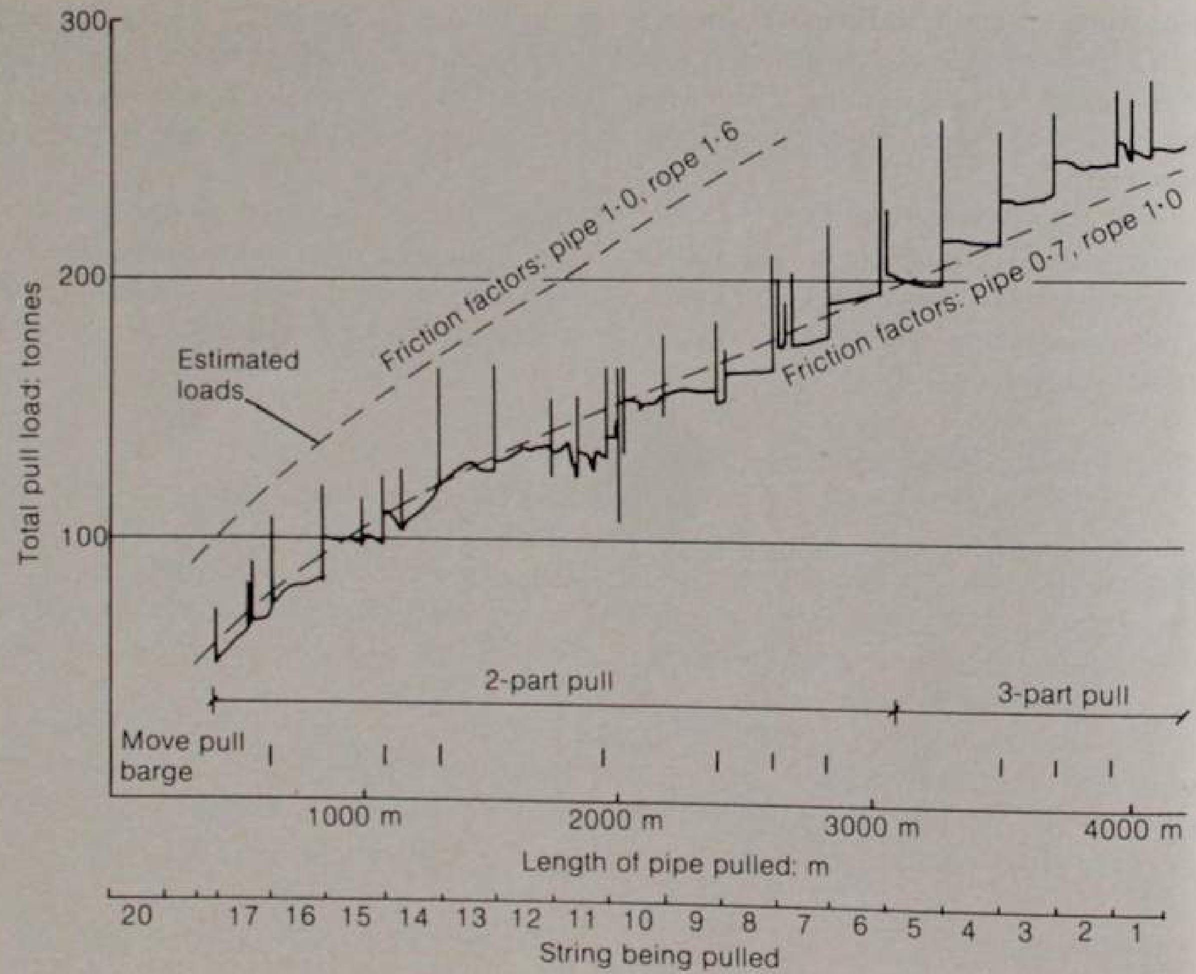


Fig. 22. Record of pull loads

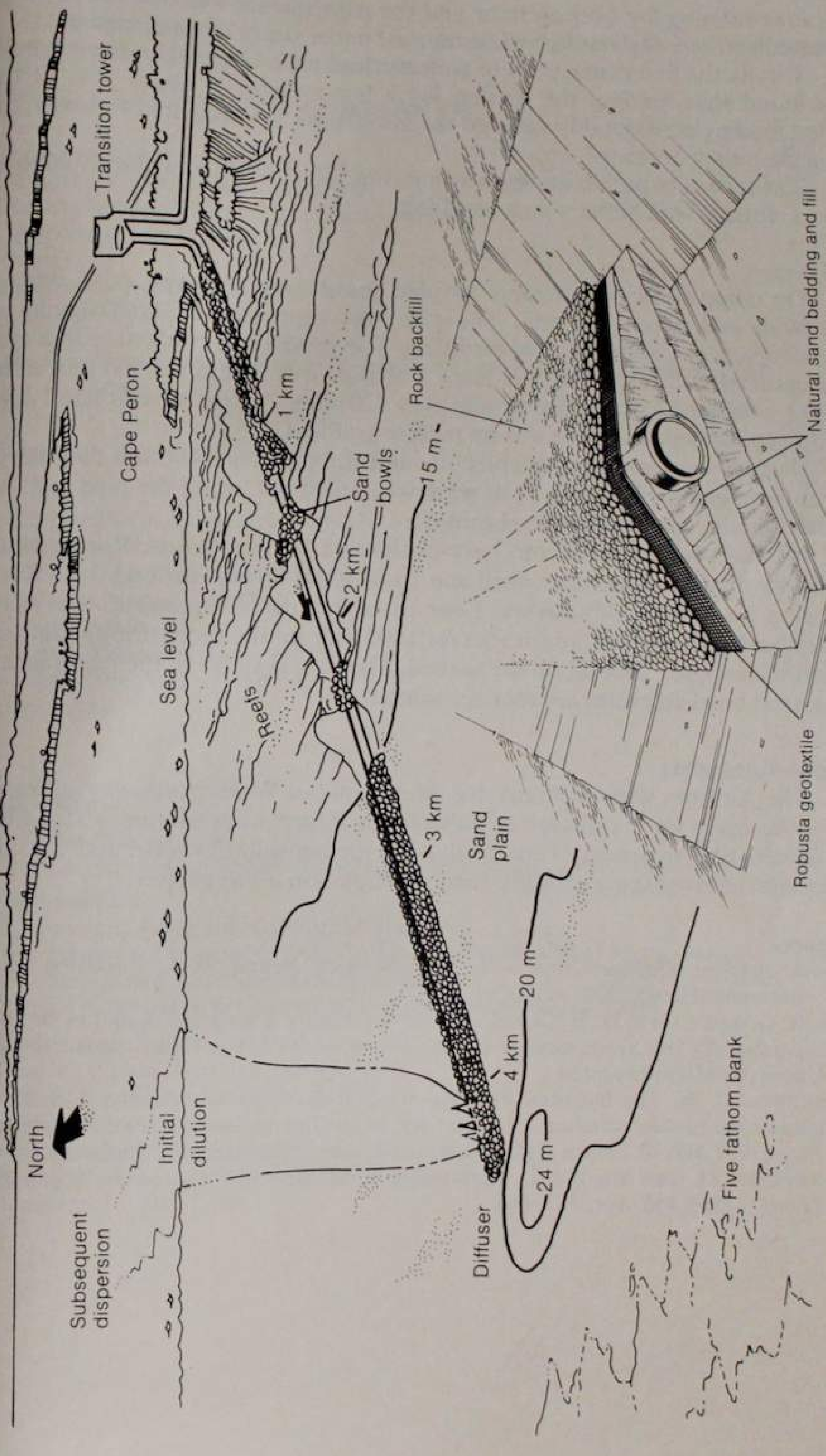


Fig. 23. (a) Schematic drawing of rock armoured pipeline; (b) typical rock armouring in trench

continuous running for over an hour and the consequence was that the bearings overheated and one day was lost while repairs 'under steam' were carried out.

59. It took the first dozen pulls to perfect winch expertise, and for the last pulls it was found that holding the tension for a few minutes at about 75% of the expected loading considerably reduced the initial peak load required to loosen the pipe on the trench bottom.

60. The record of pull loadings shown in Fig. 22 indicates how the initial peak load was reduced with better winch handling.

Backfilling

61. On completion of winching, 34 pipe spans were observed, with average length 30 m and depth 160 mm. The pipe was supported by sandbagging to reduce all spans to 25 m before water filling. The profile was checked with a spar buoy measuring rod related to tide level. The pipe was within the 500 mm tolerance, and average depth below profile was 210 mm. A final diver's hand level inspection confirmed that there was no reverse gradient.

62. Natural sand was then tremied around the pipe and Robusta 500 woven polypropylene geotextile fabric 12 m wide was rolled out along the pipe and tied to two 48 mm dia. longitudinal steel cables.

63. Rockfill was dumped from laser-positioned hopper barges. Rock placing was done by diver-directed clam shell and rock slopes formed around the diffuser ports using six contractor's divers. Four diver engineers supervised this work. Rock armouring (Fig. 23) is continuous for 1200 m from the shore through the surf zone and the exposed 1600 m on the seabed. The pipe is stable in the reef trenches but two sand bowl crossings are rock armoured.

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Appendix 1

65. Project costs are spread over 4 years from June 1980 to June 1984, and variations in exchange rates make these figures indicative only.

Feasibility study (Assuming £1 averaged A$1·80)	£ '000s	
Engineering, land and offshore works	297	
Offshore survey and site investigation	86	
Oceanography (offshore engineering and environmental) field and model running	146	
Ecology field and laboratory work	95	
	624	(IQ82)

Design and construction (June 1982–June 1984, assuming £1 averaged A$1·65)	£ '000s	
Pumping station complex: Civil	1510	
: M & E	520	
Land pipeline, including structures and oxygen injection station	12 300	
Offshore pipeline:		
Onshore pipework, transition tower	820	
Offshore work:		
excavation	2380	
pipe fabrication and installation	5260	
backfilling and restoration	2020	
Engineering	1820	
	26 630	(IQ84)

Appendix 2

Consultant's design team:
Binnie & Partners Pty Ltd in association with G. B. Hill & Partners Pty Ltd

Sub-consultants:
R. J. Brown & Associates, Offshore Engineering
W. E. Bassett & Partners Pty Ltd, – Mechanical and Electrical Engineering
R. K. Steedman & Associates, Oceanography
LeProvost, Semeniuk & Chalmer, Marine Environmental Scientists

Construction:
Pumping station complex and land pipeline:
Metropolitan Water Authority, Perth
Offshore works contract:
Leighton–Candac Cape Peron
Pipe supply, land and offshore sub-contracts:
Steelmains Pty Ltd, Perth